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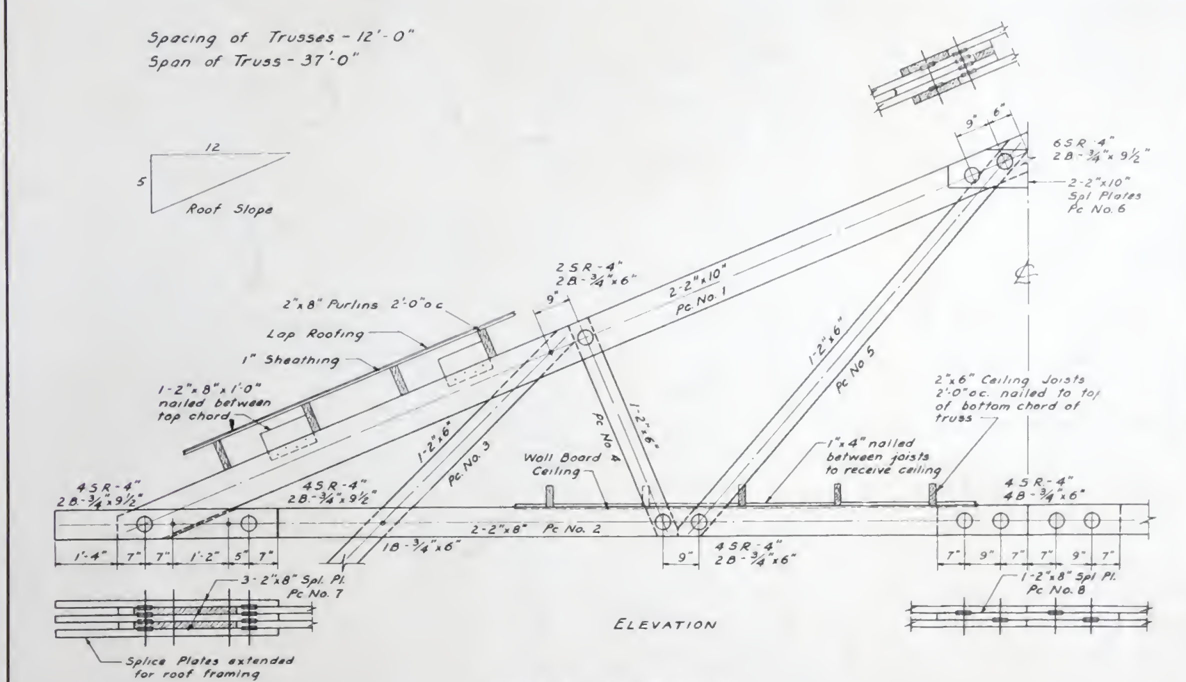
DESIGNING TIMBER CONNECTOR STRUCTURES



FINK ROOF TRUSS

TIMBER ENGINEERING COMPANY

WASHINGTON, D. C.

December 1940

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RECENT developments resulting from the advent of TECO modern timber connectors have radically improved the method of designing timber structures. New information constantly becoming available makes it practically impossible for even current engineering textbooks to present adequately this information. For this reason the Timber Engineering Company has prepared the following discussion and recommendations covering the fundamental principles of the improvements in timber design as they apply to the design and load data for TECO timber connectors given in the 1939 Manual of Timber Connector Construction.

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DESIGNING TIMBER CONNECTOR STRUCTURES

By J. E. MYER, Research Engineer*

TECO TIMBER CONNECTORS

TECO timber connectors used in structural joints are metal devices employed in the contact faces of lapped members to transfer load from one member to another, the joint being held together by one or more bolts. One-half of the connector or connector unit is installed in each contact face and transfers load proportionately to the size of the connector. It is this large bearing area of the connector against the wood near the surface of the timber where the intensity of the stress is greatest as compared to the limited area under a bolt which accounts for the high efficiency which can be developed in timber joints with connectors. With connectors in the limited joint area it is now feasible to develop up to 100% of the working stresses of the timbers. Such a high percentage of load transfer with bolts or nails alone is either expensive or impractical.

Split Ring Joint with Portion of One Member Cut Away to Show Position of Rings and Bolts

Structural lumber of the smaller dimensions, readily available in local lumber yards, may now be used with connectors to exceptional advantage, and greater efficiency is possible for large timber sizes. Furthermore, the TECO system of construction eliminates much of the complicated framing of joints found in many of the timber structures designed before connectors were available. Some of the important advantages of timber connector design are that timber designing is simplified, that more efficient use of lumber is provided and that less hardware is required, thereby resulting in more practical and more economical structures and frequently of designs hitherto thought of only in other structural materials.

* Acknowledgment for valuable assistance is made to members of the staff of the Forest Products Laboratory, particularly to J. A. Newlin and J. A. Scholten.

TYPES AND USES

Efficient connections for either timber-to-timber joints or timber-to-steel joints are provided by the several types of TECO timber connectors. The most appropriate type for a specific structure is determined primarily by the kind of joints to be made and the load to be carried. The following brief description of the functions of the different connectors is offered to assist the designer in selecting the appropriate type or types of connectors for a structure.

Fig. 1—Split Ring

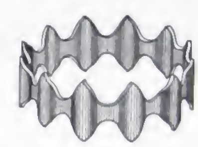


Fig. 2—Toothed Ring

Split Rings (Fig. 1) and **Toothed Rings** (Fig. 2) are two kinds of connectors used for timber-to-timber joints. Split rings carry greater loads and receive more common usage except in relatively light timber framing where toothed rings are adequate. Split rings are placed in grooves cut into the contact faces of overlapped timbers with half the depth of the ring in the groove of each member. A power tool is recommended for cutting the grooves. No groove is required for installing toothed rings since these are embedded into the wood by pressure developed by means of a high strength rod and ball-bearing washer assembly used along with a ratchet wrench or by other convenient means which may be at hand such as a power press.

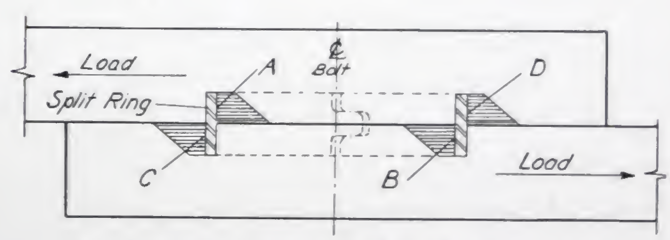


Fig. 3—Bearing Surfaces for Split Ring Connectors

The purpose of the tongue and slot split in the circumference of the ring is to permit simultaneous bearing against the core inside the ring and against the wood outside the ring. See shaded areas, Fig. 3. Due to the split in its perimeter, the ring is to some extent flexible in the direction of the load. The core inside the groove is cut larger than the diameter of a closed ring so that

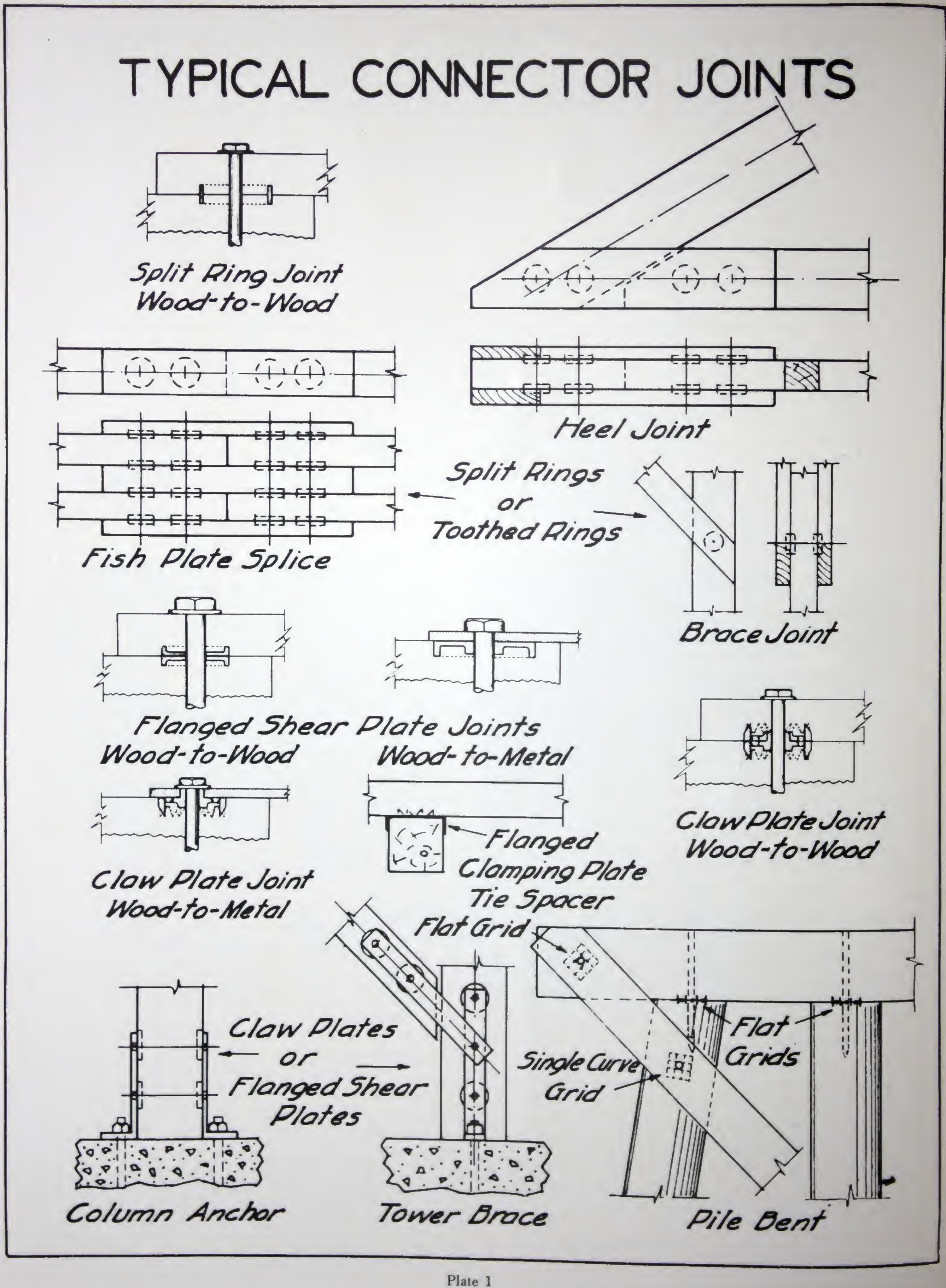

TYPICAL CONNECTOR JOINTS
Split Ring Joint
Wood-to-Wood
Heel Joint
Split Rings
or
Toothed Rings
Fish Plate Splice
Brace Joint
Flanged Shear Plate Joints
Wood-to-Wood
Wood-to-Metal
Claw Plate Joint
Wood-to-Wood
Flanged
Clamping Plate
Tie Spacer
Claw Plate Joint
Wood-to-Metal
Flat Grid
Claw Plates
or
Flanged Shear
Plates
Flat
Grids
Single Curve
Grid
Column Anchor
Tower Brace
Pile Bent

Plate 1

when the ring is installed in the two grooves of two contacting timber faces it is sprung open and fits tightly against the two cores. With wood and steel already in contact at A and B, Fig. 3, load applied on the members causes these surfaces to come promptly into bearing. A slight slip in the joint also occurs due to the width of the groove which is cut a few thousandths inches larger than the thickness of the ring to facilitate its installation. As this slip occurs, the ring is slightly elongated and comes into bearing at C and D, thus permitting bearing against the wood both inside and outside the ring.

Fig. 4—Claw Plates

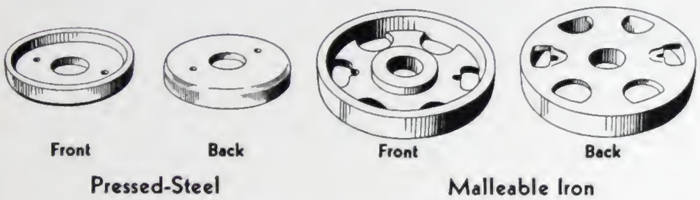


Fig. 5—Shear Plates

Claw Plates (Fig. 4) and **Shear Plates** (Fig. 5) each provide for timber-to-timber and timber-to-metal connections. In a timber-to-timber joint the following combinations of connectors may be used together, (1) one male and one female claw plate, (2) two female claw plates and (3) two flanged shear plates. In a timber-to-steel joint, all three kinds of plates may be used singly with the hole in the steel plate large enough to receive the hub of the male claw plate or the bolt to fit the female claw plate or shear plate.

These plate connectors require pre-cut daps in the timbers to receive them. Shear plate daps accommodate the entire connector, but the teeth of the claw plates must be embedded into the wood below the limit of the dap. This can be accomplished by driving or by pressure. The teeth on the claw plates hold them quite securely in the timbers during erection. Shear plates are held in their daps by nails driven through the holes or slots in the plates for that purpose. When installed, the outer hub of the male claw plate protrudes from the surface of the timber in contrast with the female claw plate and shear plate which have their flat backs flush with the timber surface. The primary advantage of the connector being flush with the timber surface is that the assembly of certain timber joints is permitted which would otherwise be impossible. The plates which fit flush with the timber surface are also more suitable for demountable structures since the joints come apart more readily and there is less chance for the connectors to be pried out of their daps. The smaller amount of slip in claw plate joints as compared with shear plate joints may make the use of claw plates more desirable for some types of joints, especially where reversal of stresses may occur.

Spike Grid connectors (Fig. 6) are designed for timber-to-timber joints in which the contact faces are either flat or curved. The flat grid is used to join two flat faces of lapped timbers; for example, timber braces to timber posts in trestle bents. The single curve grid is used to join a member with a flat surface to one with a curved surface, such as a timber brace to a round pile. The double curve grid is used to join two parallel members with curved surfaces, for example, two round piles. Spike grids are embedded into the timbers by pressure most conveniently developed by a high strength rod and ball-bearing washer assembly. These connectors give a high degree of rigidity to joints with an exceptionally small amount of deformation under working loads.

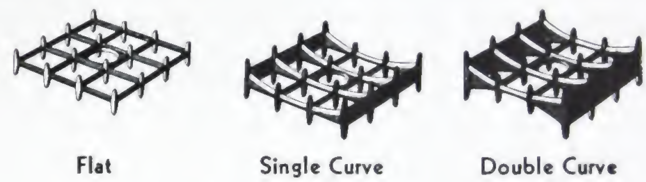


Fig. 6—Spike Grids

Fig. 7—Clamping Plates

Clamping Plates (Fig. 7) are suited primarily for connecting timbers lapped at right angles, for example, as "tie spacers" for fastening ties to timber guard rails on open deck railroad trestles or bridges. The plain clamping plate makes a somewhat more rigid connection in that the teeth are embedded into both members whereas the flanged plate may permit a slight movement. On the other hand, the flanged plate permits greater ease in the replacement of ties. The installation of the plates requires that the teeth be embedded by means of driving. Plain clamping plates with teeth on two sides take two driving operations, one to get the teeth into the tie and another to get the teeth on the opposite face into the timber guard rail. The flanged clamping plate with teeth on one side requires only one driving operation, that of seating the teeth into the timber guard rail.

DESIGN INFORMATION

The TECO system of construction follows the same general structural design procedure as other materials

and methods. It is the simplification of the joints by using timber connectors which makes possible greater efficiency in the use of lumber and ease of designing than heretofore. The necessary information required for this system of construction is given in the list of Supplements to Wood Structural Design Data published by the National Lumber Manufacturers Association:

Supplement No. 1 Working Stresses for Structural Lumber and Timber

Supplement No. 2 Bolted Wood Joints, Safe Loads on Common Bolts

Supplement No. 4 Wood Columns, Safe Loads

Supplement No. 5—Wood Trusses, Strength Coefficients, Length Coefficients, and Angles.

Supplement No. 6 Timber Connectors, Design and Load Data, also known as the "Manual of Timber Connector Construction" or the "Manual".

These supplements are available on request either from the Association or from the Timber Engineering Company, a subsidiary, both concerns with addresses at 1337 Connecticut Avenue, Washington, D. C. The following description of terms, discussion of rules and examples as they apply to connector construction are presented to assist the designer in the application of this information to specific cases.

DESCRIPTION OF TERMS RELATING TO CONNECTOR DESIGN

The descriptions of terms used in the Manual are given as they pertain to timber connector design.

Timber Connectors are metal devices employed in timber joints to transfer load from one member to another, the members being held together by one or more bolts.

Standard Design Loads or working loads for connectors tabulated in the Manual apply to most of the loading conditions under which all types of TECO connectors are used where the dead load is less than the live load, and where the full live load is applied for relatively short periods of time, such as a snow load on a roof truss. Standard Design Loads are 115% of basic values.

Basic Values are derived by applying appropriate factors to test data. They allow for a permanently applied full load, a condition not usually encountered in the use of timber connectors.

Species Group is a classification given different structural species of lumber based on the relative loads developed by connectors in those species.

The Compression Side of a connector is the outside portion of the connector which is in compression against the wood.

Shear Area for a connector is considered as being equal to that area between the compression side of the connector and the edge or end of a member, or between two connectors and of a width equal to the diameter of the connector. (See Fig. 9-B.)

Angle of Load to Grain is the angle formed by the direction of the load transmitted to a member by a connector and the direction of the grain in the member. (See Fig. 8.)

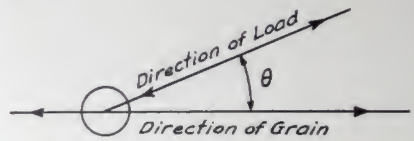


Fig. 8—Angle of Load to Grain

End Distance is the distance measured parallel to the grain from the center of the connector to that point toward the end of the member which provides a shear area for the connector equal to that provided by an end cut square across the member the same distance from the center of the connector.

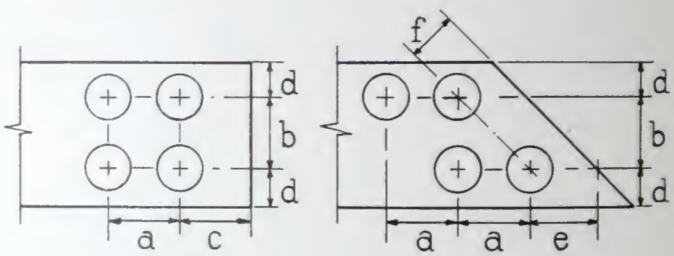


Fig. 9-A—Method of Measuring Spacings and End Distances for Timber Connectors with Square or Single Diagonal End Cut

a. Connector spacing parallel to grain
b. Connector spacing perpendicular to grain
c. End distance
d. Edge distance
e. Maintain end distance as minimum
f. Maintain edge distance as minimum

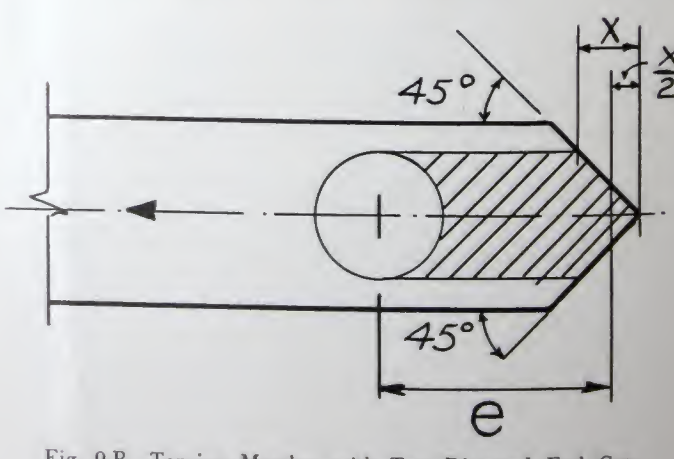


Fig. 9-B—Tension Member with Two Diagonal End Cuts

For the above case with the two diagonal cuts at 45°, the end distance (e) is measured to a point equal to one-half the distance from the end of the member to a line perpendicular to the axis of the member passing through the intersection of the diagonal cut and the line projected from the edge of the connector.

End distance is measured from the center of the connector parallel to the grain to the end cut where this is square across the member or where a single diagonal cut extends across the full width of the projected diameter of the connector. If a single diagonal cut does not extend fully across the projected diameter of the connector, or if two diagonal cuts are made, the shear area must be determined to find the end distance. This can usually be estimated reasonably close for design purposes. (See Figs. 9-A and 9-B.)

Edge Distance is the distance measured perpendicular to the grain from the center of the connector to the edge of the face of the piece into which the connector is installed.

Edge Distance on the Compression Side of the connector is measured from the center of the connector to the edge of the piece nearest the center of the compression side of the connector. (See Fig. 10.)

Edge Distance Opposite the Compression Side of the connector is measured from the center of the connector to the edge of the piece nearest the center of the outside portion of the connector not in compression against the wood.

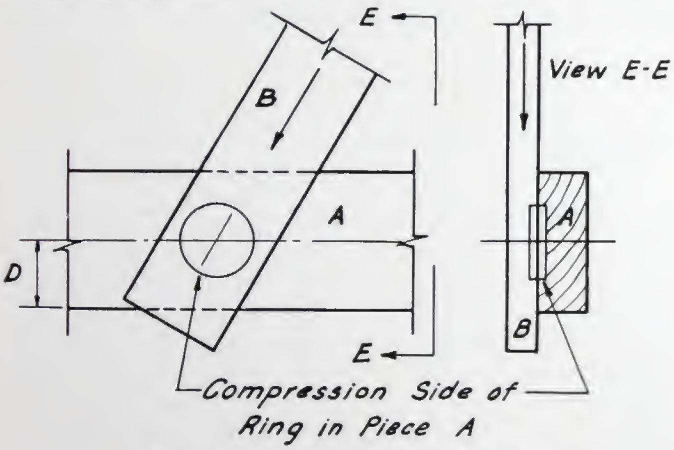


Fig. 10—Compression Side of Ring

D—Edge distance on compression side of connector in piece A which is assumed to be held in place by forces not indicated.

Spacing of Connectors is the distance between the centers of two connectors in the same timber face. Connectors are spaced either parallel or perpendicular to the grain.

Connectors are considered to be spaced *parallel* to the grain when a line drawn through the axes of connectors in the same timber face forms an angle less than 30° with the direction of grain. Connectors are considered to be spaced *perpendicular* to the grain when a line drawn through the axes of connectors in the same timber face forms an angle of 30° or more with the direction of grain.

Standard Spacings and Distances are those required for full Standard Design Loads. Greater spacings or distances do not permit increases in connector capacities.

Minimum Spacings and Distances are the smallest permitted for use with connectors and except where minimum and standard spacings and distances are the same, require reductions in connector loads as specified in the Manual.

Cross- or Stitch-Bolt is a bolt located halfway from the edge of the connector to the end of a member, extending in a direction parallel to the face in which the connector is located and at right angles to the grain. (See Fig. 11.) Its application to connector design is in tension members as it permits end distances for tension members to be equal to the lesser standard end distances for compression members and still carry full rated loads.

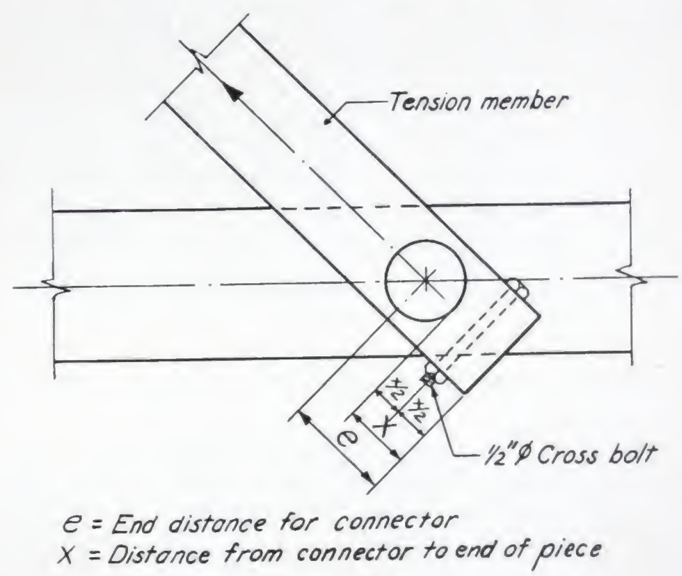


Fig. 11—Illustration Showing Use of Cross Bolt

CONDITIONS AFFECTING CONNECTOR LOADS

Angle of Load to Grain

Wood has greater compression strength parallel with the grain than across the grain and, in general, the capacities of connectors decrease as the angle of load to grain increases. It is important, therefore, to determine the member in which the load acts at an angle to grain. This may readily be determined for each member by considering the forces acting and the relative position of the overlapped members in the joint.

Fig. 12-A shows a two member joint with members entering at an angle θ. If the vertical component of diagonal member A is counteracted by reaction C then the direction of load on the connector in member B must be parallel to grain in this piece since the only force applied is axial. Member A, however, in addition to its axial load, has its vertical component counteracted by reaction C so that only the horizontal component remains,

resulting in a load on connector in member A equal and opposite in direction to that in member B. It is this direction of load on the connector with reference to the direction of grain in each member which determines the angle of load to grain for each member. On the other hand, if horizontal member B is held in equilibrium by reaction C, then the direction of load on the connector in member B is at an angle of load to grain equal to θ and the direction of load on the connector in member A is parallel to grain. From this it is apparent that a member loaded only axially bears on the connector in a direction parallel or 0° to the grain, also that a member with forces acting on it other than axial bears on the connector at an angle to grain.

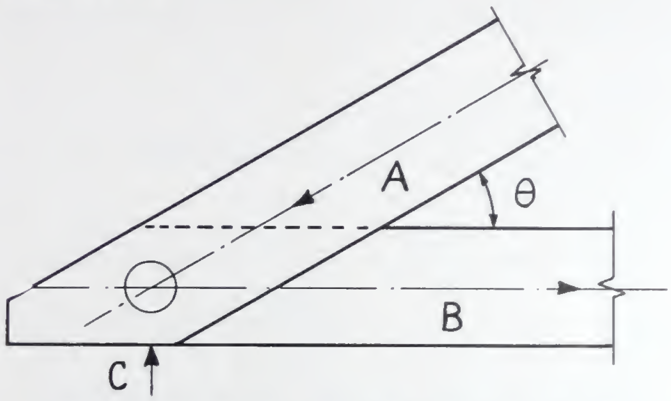


Fig. 12-A—Two Member Joint. See Discussion in Text.

Assuming further that member A is supported by reaction C, and that member B has a superimposed load by the addition of ceiling joists spaced between panel points, then the direction of the load on the connector in member B is the resultant of the combined axial load and the reaction of the superimposed loads. The direction of this resultant and the direction of grain in each piece is used in determining the angles of load to grain in members A and B.

An analysis of the three member joint in Fig. 12-B with three members entering the joint from different directions shows that each of the two side pieces has only an axial load, while the center member has two forces acting on it, the vertical components of which counteract each other, similar to the vertical component of member A and its reaction C in Fig. 12-A. In this case, however, the forces are applied to connectors in the center member in the directions of the grain in the side members and the reactions in the center members are equal and opposite in direction. Therefore the angle of load to grain in the center member is determined by the angle formed by the center member and each side member. It will be noted that in a three member joint it will normally be the center member in which the direction of load on the connector is at an angle to grain.

Likewise in a joint with five overlapped members entering from three directions, the second and fourth members, counting from one side to the other, are usually those loaded at an angle to the grain.

Center member, B. Fig. 12-B, has the loads applied at an angle greater than 30° and toward opposite edges, a condition which requires that both edge distances must be equal to that specified for the compression side of the connector. The angle of load to grain is 0° in members A and C and, therefore, the capacity of the connectors in these pieces is greater than in member B. The load capacities for connectors are limited by the angle of load to grain in piece C, consequently end distances of members A and C may be reduced according to the percentage reduction allowed with reduced loads.

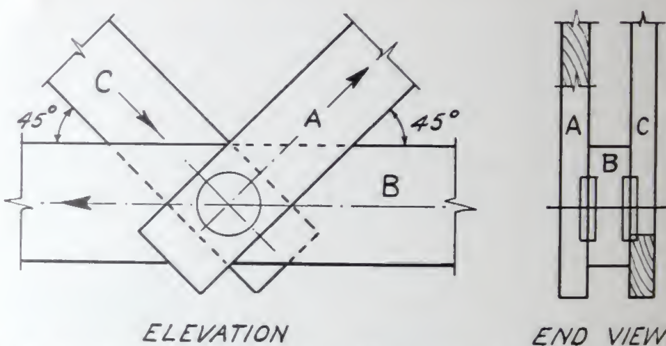


Fig. 12-B—Three Member Joint with Load at Angle to Grain in Center Member; Load Parallel to Grain in Both Side Members

End Distances and Connector Loads

Full connector loads for 0° angle of load to grain require standard end distances. Connector loads less than those for 0° angle of load to grain may take sub-standard end distances, or where sub-standard end distances are used, connector loads must be reduced proportionately. Permissible connector loads for sub-standard end distances are determined by interpolation between standard and minimum values. For influence of a cross-bolt on end distance in tension members, see above. A cross-bolt placed in a member which is not seasoned to the extent it will reach in use should be tightened again after the member is thoroughly seasoned.

Edge Distances and Connector Loads

Full connector loads for the different angles of load to grain require standard edge distances. Where the angle of load to grain is less than 30°, the standard edge distance is also the minimum edge distance and must be maintained. For an angle of load to grain greater than 30°, the edge distance must be increased by the amount specified on the compression side of the connector to secure the rated load. If the edge distance is not increased as specified, when the angle of load to grain is

greater than 30°, the connector load is reduced; the maximum reduction being 15%. Permissible connector loads for sub-standard edge distances may be determined by interpolation between standard and minimum edge distances. The 30 degree angle for differentiating edge distances as well as for spacings discussed later, while based on tests, has been arbitrarily selected to present a convenient rule for the two general classes of loading, namely parallel and perpendicular to grain. It is to be expected that engineers in preparing designs, when necessary, will exercise their judgment in arriving at what seems to be reasonable values in borderline cases around the 30° angle of load to grain.

Spacing and Connector Loads

Full connector loads for 0° angle of load to grain require standard spacings. With spacings less than standard, the design load capacity of the connectors in a row must be reduced. One connector in the row is excepted from the reduction in the convenient rule which has been devised for determining the reduction of load capacity for reduced spacing. This rule is based on test results and while not intended to designate the actual proportion of the total load carried by each connector in a row, it accomplishes the result of a proper reduction of load for the row of connectors as a whole. The one connector in the row excepted from reduction in load capacity due to reduced spacing is considered, for the purpose of computing the total load capacity of the connectors, as carrying 100% of its rated load. For minimum spacing of connectors parallel to grain a 50% reduction, and for minimum spacing perpendicular to grain a 15% reduction, applies to the load capacities for the balance of the connectors in the row. Reductions in connector loads due to angle of load to grain from 0° to 30° may take reduced spacings to conform to the reduced loads. If spacings are reduced below this amount connector loads must then be reduced proportionately.

Sub-Standard Distances, Sub-Standard Spacings and Connector Loads

Connectors in a joint, in addition to sub-standard spacings, may have sub-standard end distances when the angle of load to grain is 0°— 30°; or they may have both sub-standard end and sub-standard edge distances when the angle of load to grain is 30°— 90°. To find the load capacity of a joint loaded 0°— 30° with two or more connectors spaced in a row with spacing and end distance both sub-standard, the procedures previously described apply; however, for convenience in computing the capacity of the joint it is assumed that the end connector is the connector which would carry 100% capacity with full end distance, but since it has a sub-standard end distance this 100% capacity is reduced for the sub-standard end distance. This same principle of reduced loads for joints applies when angle of load to grain is 30°— 90°, but even though the end connector might have both sub-standard end distance as well as sub-standard edge distance, the reduction applied to the 100% capacity of the end connector is only the greater one of the two reductions.

Spacing of Connectors in Members Overlapped at an Angle

When two members come together to form a joint and the angle of load to grain is less than 30°, the connectors may be spaced with reference to the grain in either piece or in any intermediate position which may be convenient. Furthermore, if three members come together to form a joint with the angle of load to grain less than 30° for any two contacting members, as may be found in the peak joint of a Fink truss (See Fig. 13) with top chord B, web member A, and horizontal splice C, the connectors may be located with reference to any member and still develop the rated connector loads in each of these three members.

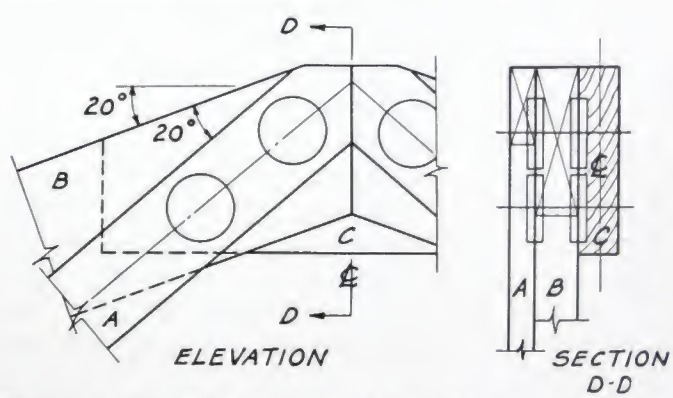


Fig. 13—A Three Member Joint with Each Two Contacting Members Forming an Angle of Load to Grain Less than 30°

Where the overlapped members of a joint enter at angles greater than 30°, the connectors must be spaced with reference to the member in which the angle of load to grain is greatest. If each of two overlapped members are loaded at an angle greater than 30° due to superimposed loading, the connectors must have perpendicular to grain spacing, which is greater than the spacing parallel to grain; in this case the connectors may be spaced with reference to either member or adjusted to any position most suitable for the assembly of the joint. Furthermore if 85% or less of the rated load on the connectors is developed, the connectors may take the parallel to grain spacing.

Wind, Earthquake, Net Section, etc.

Other factors influencing connector loads are wind, earthquake, impact, lumber thickness, and condition of lumber. These factors and the required net section of a member in a connector joint are discussed in the Manual of Timber Connector Construction and are therefore not included here.

FACTORS INFLUENCING CONNECTOR JOINT DESIGN

Selection of Lumber Species and Sizes

Lumber species selected for a structure will depend on the loads to be carried and the relative cost and availability of the material. In large structures, where it is necessary to carry heavy loads, a structural species with relatively high working stresses and high connector working loads will generally be found to be most satisfactory. However, if other considerations such as availability and cost of material are in favor of other species, these should also be considered.

The TECO system, because of the required overlapped contact faces for placing connectors, utilizes to advantage thinner lumber than do other types of timber construction. Another reason that thinner sections may be used economically and efficiently as compression members, is due to the fact that when the members are secured near their ends with connectors, the allowable load for a given l/d is increased according to the spaced column principle developed by the U. S. Forest Products Laboratory. (See Wood Structural Design Data Supplement No. 4, Wood Columns-Safe Loads.) The thickness of members should be selected with due consideration for the load to be carried, which involves the l/d ratio if the member is in compression, and for the overlapped area necessary to afford adequate end or edge distance and spacing for connectors. These limits will soon become apparent when proceeding to design a joint for a structure.

Selection of Connector Type and Size

The structure to be designed will quite definitely dictate the type or types of connectors to be selected. When the joints involve only wood members, one of several types of connectors might be used. However, since each connector is designed for a quite specific purpose as previously described, its efficiency is dependent on the type of connection and the amount of load to be carried. Timber-to-timber joints in structures carrying heavy loads may employ split rings, shear plates or claw plates. Split rings are used only in timber-to-timber joints since they must fit into grooves in the faces of the members joined; shear plates or female claw plates may be used in either timber-to-timber or timber-to-metal joint, because they lie flush with the surface of the member. Frequently, construction details are such that a member must be slid into place between other members or metal plates and it is this type of connection where the flush type of plate is required.

In a structure designed for relatively heavy loads, where frequent dismantling is not anticipated, split rings will be found more appropriate than either shear plates or claw plates because of their lower cost. On the other hand, if a high percentage of the joints in the structure are of the wood-to-metal type, shear plates or claw plates are necessary in these joints to make an efficient connection. For this latter condition, rather than having two types of connectors in the structure with split rings for timber-to-timber joints and plate connectors for timber-to-metal joints, it may be desirable to use the plate type connector throughout. Should a proposed structure be of a type to be dismantled several times, such as an oil derrick, the shear plate connector will be found advantageous. These connectors are installed flush in the timber and are held in place by nails. When the joints are taken apart there is no possibility of impairing the strength of the joint by mutilating the wood around the connector, and the flush plates provide for compact piling of members.

Timber structures carrying relatively light loads and with thin structural members such as trussed rafters have employed toothed rings to very good advantage. No grooving is necessary and with templates for holding the members during assembly, toothed rings are installed rapidly with the high strength rod and ball-bearing washer assembly or by means of presses rigged up on the job for the purpose.

Another factor which may influence connector size is that frequently a single size is most efficient for nearly all the joints in a structure. Then, rather than specify two or more sizes, it is generally desirable to use the common one for the remaining joints. Two or more sizes should not be avoided, however, where it is evident that a single size will not serve effectively and economically.

PROCEDURE FOR THE DESIGN OF CONNECTOR JOINTS

For the convenience of those not already familiar with the customary procedure employed in the design of timber joints, the following recommendations are offered. While this procedure will apply to most structures, special cases may call for variations, but with the fundamental steps in mind the variations can easily be made.

1. Determine stresses for structural members.
2. Form a tentative plan for framing the structure and select the lumber species to be used.

3. Compute the sizes of the structural members.
4. Select the type and size of connectors which seem to be most suitable for the structure.
5. Determine the angle of load to grain for each member in the joint to get ring capacity, required edge and end distances and spacings for connectors.
6. Design those joints first which carry the greatest loads, since the sizes of the members required to space the connectors in these joints will be a determining factor in arriving at final member sizes.
7. Determine the amount of load in the joint which must be transferred between each two members to be joined and design for the largest load first; the positioning of the connectors for the smaller loads will usually not be a problem; they should, however, be checked.
8. Compute the number of connectors required to carry the loads and proceed to locate them on gauge lines drawn with reference to the edges of the overlapped members. The spacings between connectors must also be checked.
9. If gauge lines do not permit full spacings and full capacities of connectors must be developed, the face widths of certain members must then be increased to provide the spacings required, or it may be that other sizes of connectors can be used without increasing the lumber sizes.
10. Check end distances and spacings for connectors in each member.

An example illustrating the steps when designing structural members in a specific truss and the design of several typical joints will perhaps most clearly bring out the several fundamental principles previously discussed. Their application to the design of other structures and joints will be quite apparent. For this purpose a 50 foot Fink roof truss has been selected and a diagram for one-half of the truss with loads for each member is shown in Fig. 14. It is assumed that the roof loads are applied at the panel points.

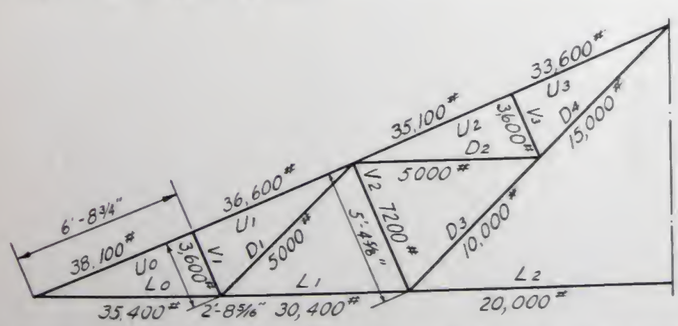


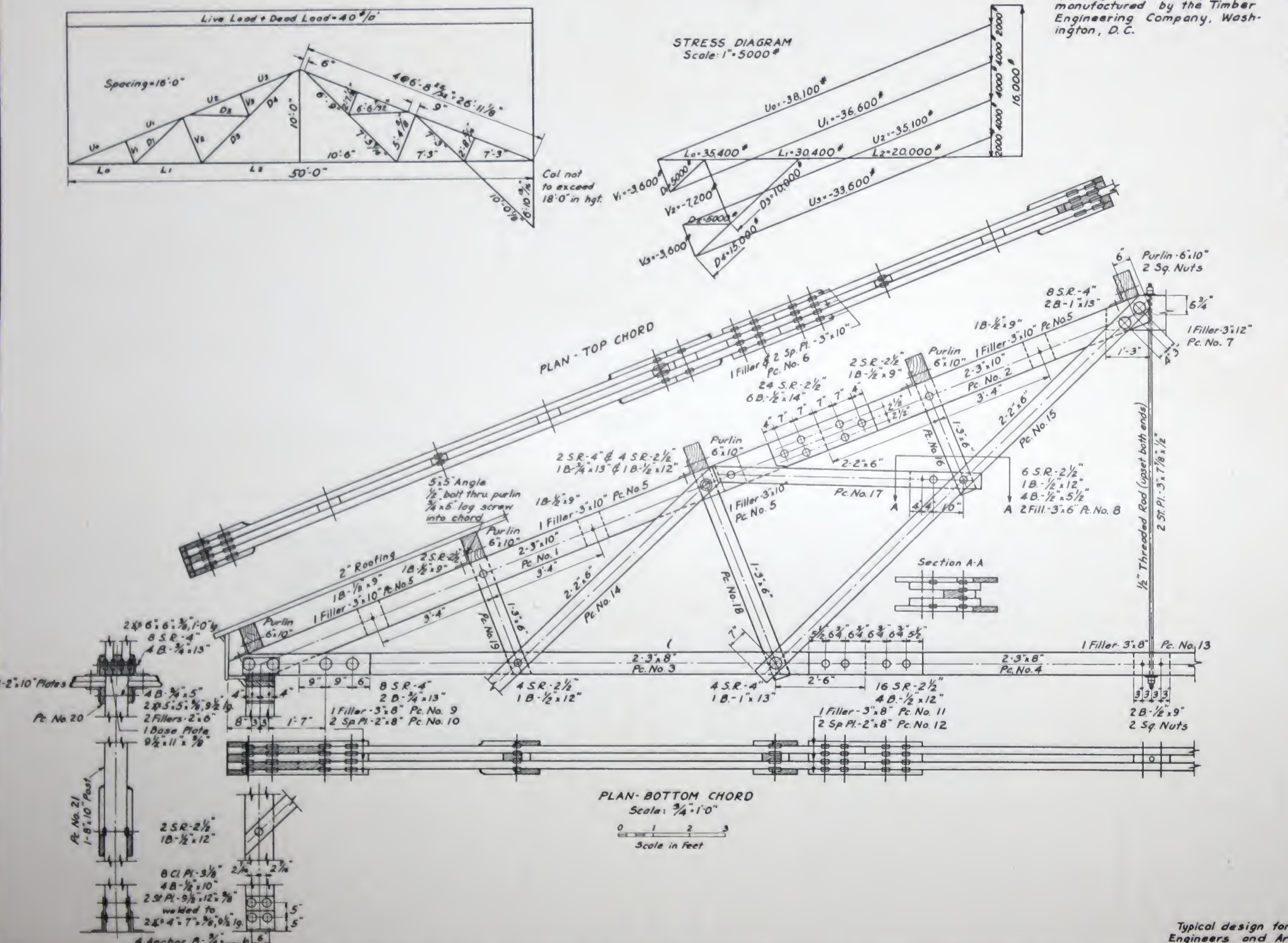
Fig. 14—Diagram for One-half of 50 foot Fink Roof Truss Giving Loads for Members with Spacing of Trusses 16 feet on Centers and Combined Live and Dead Loads of 40 lbs. per Square Foot

DESIGN OF STRUCTURAL MEMBERS

Top Chord—Since the greatest load is developed in the top chord panels (see Supplement No. 5 Wood Trusses), the structural sizes should be designed for these loads first. To determine the size of the top chord in Fig. 14 it is assumed (1) that the chord will be composed of two members rather than one thereby providing greater surface area for placing connectors in the overlapped joint members, (2) that the members will be joined with connectors at the panel points to make it possible to take advantage of greater working stresses in compression due to the spaced column principle (see Supplement No. 4 Wood Columns), and (3) that lateral support will be provided at each panel point to prevent lateral buckling of the top chord.

When computing the sizes of spaced column members as for the top chord in question, it will usually be found that a large value for "d" in the l/d ratio (l = length of panel in inches, d=least thickness in inches) is most suitable up to the point where the width of the member becomes too narrow to permit the placement of the necessary number of connectors. If the value for "d" becomes too small, greater overlapped area results but the working stresses of the lumber are rapidly reduced and the members become inefficient and uneconomical.

Reference to the top chord of the 50′ Fink roof truss, Fig. 14, shows a panel length of 6′ 8¾″ and a maximum load of 38,100 lbs. With members 1⅝″ thick, the l/d ratio for the 6′ 8¾″ panel length is 50; with lumber 2⅝″ thick, the ratio is 31; and with lumber 3⅝″ thick the ratio is 22. Then, for lumber with a modulus of elasticity (E) of 1,600,000 lbs. per square inch and a compressive stress (c) of 880 lbs. per square inch, the working stresses for the different l/d ratios will be 440 lbs., 820 lbs., and 880 lbs. per square inch respectively. (See Supplement No. 4, page 19, curve C for Long Time Loads for Spaced Columns with connections located within l/20 from the end. With c equal to 880, the curves on page 19 for c=900 may be used since there is such a slight difference; no value, however, for a c=880 grade should exceed this amount.) Dividing the 38,100 lb. load by the values for the different l/d ratios, the 1⅝″ thickness requires 87 square inches, the 2⅝″ thickness requires 47 square inches and the 3⅝″ thickness requires 43 square inches. The 1⅝″ thickness is evidently impractical since two chord members 28″ wide would be necessary, also chord members 3⅝″ thick take only a 7½″ width to carry the load but do not furnish sufficient surface area to accommodate the connectors. Members 2⅝″ thick require a 9½″ face width to carry the load and provide sufficient overlapped area for placing connectors, and are therefore recommended for the top chord.



TIMBER CONNECTORS

Connectors shall be TECO split rings and claw plates as manufactured by the Timber Engineering Company, Washington, D.C.

CAMBER

The proper camber may be introduced into this truss by raising the lower chord 1" at the center during fabrication.

LUMBER

Lumber shall be of a structural grade with minimum allowable working stresses in lbs. per sq. in. as follows:

880# Compression parallel to grain.
1200# Extreme fiber in bending.
1,600,000# Modulus of elasticity.

Allowable unit working stresses for commercial grades of lumber are given in the National Lumber Manufacturers Association's leaflet, "Working Stresses for Structural Lumber and Timber," or are given in the Grading Rules available from the Regional Lumber Manufacturers Associations.

MATERIALS LIST PER TRUSS

LUMBER CUTTING BILL (S4S)

Mk.	Size	Length	Make	Cut From	Order	F.B.M.
1	3"x10"	18'-0"	4	18'-0"	4	180
2	3"x10"	10'-0"	4	20'-0"	2	100
3	3"x8"	15'-11"	4	16'-0"	4	128
4	3"x8"	16'-0"	2	16'-0"	2	64
5	3"x10"	0'-8"	8	12'-0"	2	60
6	3"x10"	3'-0"	6	Pc. No. 5		
7	3"x12"	2'-6"	1	4'-0"	1	12
8	3"x6"	2'-0"	4	18'-0"	2	54
9	3"x8"	4'-0"	2	16'-0"	1	32
10	2"x8"	4'-0"	4	16'-0"	1	22
11	3"x8"	3'-2"	2	Pc. No. 9		
12	2"x8"	3'-2"	4	14'-0"	1	19
13	3"x8"	1'-0"	1	Pc. No. 9		
14	2"x6"	18'-6"	4	20'-0"	4	80
15	2"x6"	15'-4"	4	16'-0"	4	64
16	3"x6"	3'-5"	2	Pc. No. 8		
17	2"x6"	7'-7"	4	16'-0"	2	32
18	3"x6"	6'-4"	2	Pc. No. 8		
19	3"x6"	3'-7"	2	Pc. No. 8		
20	2"x6"	0'-9½"	4	Pc. No. 14		
21	8"x10"	18'-0"	2	18'-0"	2	240
					Total F.B.M.	1087

HARDWARE

No.	Item	Size
60	Split Rings	4" φ
16	Claw Plates	3⅛" φ
8	Machine Bolts	½" x 5½"
12	" "	½" x 9"
8	" "	½" x 10"
16	" "	½" x 12"
12	" "	½" x 14"
8	" "	¾" x 5"
14	" "	¾" x 13"
6	" "	1" x 13"
100	Plate Washers	2" x 2" x ⅛"
24	" "	3" x 3" x 3/16"
16	Cut Washers	2" φ
4	Angles	6" x 6" x ⅜", 1'-0" lg.
4	"	5" x 5" x ⅜", 9½" lg.
4	" welded to	4" x 7" x ⅜", 9½" lg.
4	Steel Plates	9½" x 12" x ⅜"
2	" "	3" x 7⅞" x ½"
2	" "	11⅛" x 9½" x ⅜"
120	Split Rings	2½" φ
8	Anchor Bolts	¾" —— lg.
1	Threaded Rod ½" x 11'-3" threaded 3" each end.	

Typical design for use of Engineers and Architects

TIMBER ENGINEERING COMPANY
WASHINGTON, D. C.

FINK ROOF TRUSS
Span-50'-0" Rise-10'-0"

SCALE 3/4" = 1'-0" SHEET 1 OF 1

DATE
DESIGNED BY J.H.Carr 12/12/34
CHECKED BY P.Landsem 3/19/35
TRACED BY R.E.Harter 11/5/39

DRAWING NO. 35

Revised N.M. 10/30/39 J.H.C.

PLATE II

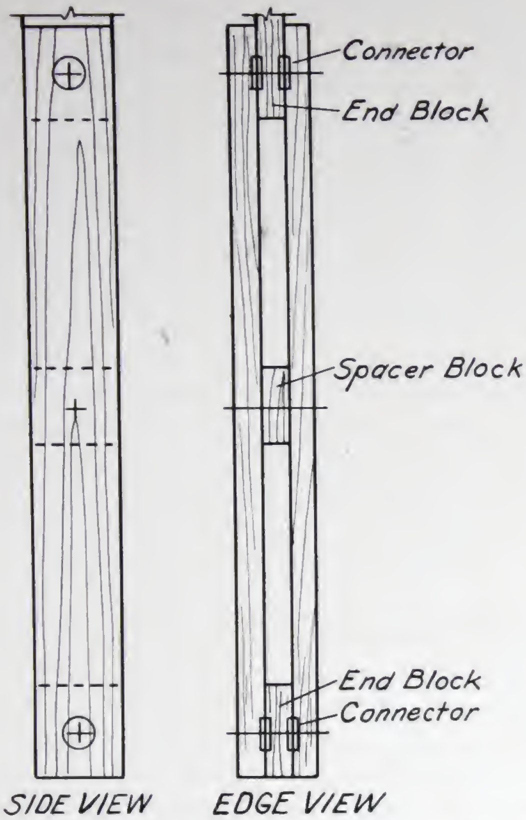


Diagram of Two-Member Spaced Column (Connector Joined). See Supplement No. 4, Spaced Columns—Safe Loads, for details covering size and location of connectors, end blocks and spacer blocks.

Bottom Chord—The bottom chord member L_0, Fig. 14, has a tension load of 35,400 lbs. Assuming a lumber grade of 1,200 lbs. per square inch in tension or extreme fibre in bending (f), 29.5 square inches of cross section are required. This is equivalent to approximately two pieces of 3″ x 6″ with a total net section of 29.54 sq. in. (See back cover of Manual.)

Three inch thickness of members is selected primarily to maintain the same thickness as in the top chord members so that the splice plates will lie flat on the faces of the abutting upper and lower chord members. The 6″ width of lower chord members is sufficient to accommodate the 4″ split ring connectors in this joint, but due to the necessity of an 8″ width to develop the load at an angle to grain in another bottom chord joint for this truss, the bottom chord is made 8″ wide throughout its entire length, thereby eliminating an extra splice joint in the bottom chord which would otherwise be necessary.

Web-Members—Web-members in compression can best be handled as solid columns, with a relatively small l/d ratio, in contrast to web-members in tension, which may be comparatively thin since their l/d is not a factor in their working stresses. This combination provides for efficient design and a symmetrically loaded joint by placing the compression members between the double chord members and the two tension members on the outside. Compression member V2 with a greater load than occurs in either V1 or V3 is tentatively considered as having a 3″ thickness (2⅝″ net) to correspond to the appropriate 3″ thickness of the center splice plates for making splices in the chord members. (The thickness of the center member of three splice plates used for joining a two member chord should normally equal half the total thickness of the two members comprising the chord; the two outside splice pieces, making up the remaining necessary section.) Using the same 880 lb. c grade of lumber as in the top chord and with an l/d ratio of 25, the allowable working stress for member V2 is 680 lbs. per square inch. (See Supplement No. 4, page 19, Curve A for Long Time Loading of Simple Solid Columns.) The load of 7,200 lbs. requires 10.6 square inches of cross section. A 3″ x 6″ piece with a cross section of 14.77 square inches will accommodate this load and provide a 6″ face for connectors.

Assuming a 2⅝″ net thickness for compression members V1 and V3, Fig. 14, corresponding in thickness to member V2, the l/d ratio is 12.3 for each of these two members. In 880 lb. c grade of lumber, this gives a working stress of 880 lbs. per square inch, and for the 3,600 load, approximately four square inches of cross section are required. A 3″ x 4″ piece with a net cross section of 9.52 square inches is the smallest size possible, since the 2⅝″ thickness must be maintained to fit between the chord members and a 3⅝″ width of face is necessary for a 2½″ split ring; therefor this member size is recommended.

Tension members D3 and D4 with a 15,000 lb. maximum load and with the 1,200 lb. f grade of lumber used in the truss, will require 12.5 square inches of cross section. Two 2″ x 6″'s with a total of 18.2 square inches of cross section will provide more section than necessary. The other tension members, D1 and D2, with a load of 5,000 lbs. each, need less section than member D4, but 2″ x 6″'s are recommended to keep the sizes uniform and further because smaller structural members are not generally recommended.

DESIGN OF TIMBER JOINTS

The order in which the joints in the Fink Truss, Fig. 14, might logically be designed would be first the heel joint, because of the large loads to be transmitted, then the peak joint with relatively large loads and with members entering the joint from three directions, and finally the remaining joints in any order desired since they are comparatively simple. However, to more clearly present the design principles which apply to timber connector joints, a few typical joints will be discussed first, and then the heel and peak joints of the Fink Truss, Fig. 14.

Load Parallel to Grain

Tension Joint—In the tension joint loaded parallel to grain, Fig. 15, consisting of two overlapped members and split rings placed in the contact faces, assume that the load to be carried is 14,000 lbs. and that the lumber is 1200 lbs. f grade of Group B Species.

Design the joint for the following conditions:

1. Full end distances and spacings for connectors.
2. Permissible reduced spacings.
3. Permissible reduced end distances.
4. Permissible reduced spacings and end distances.

The four conditions outlined are presented to show the different combinations possible and how sub-standard spacings and distances may be computed for reduced loads. *In actual design, it is recommended that reductions for spacings and distances be made on both, rather than on spacings or distances alone and thereby maintain more uniform load reductions for all connectors.*

1. Full End Distances and Spacings

Lumber Size: The 1200 lbs. f grade of lumber requires 11.7 square inches of section to carry the 14,000 lb. load. In the table on the back cover of the Manual, it will be found that a 2″ x 8″ member with a sectional area of 12.19 square inches and a 3″ x 6″ member with a sectional area of 14.77 square inches, are the smallest nominal lumber sizes that will carry the load in tension.

Connector Size: Since split rings are specified for the joint, either one or two rows of 2½″, or 1 row of 4″ split rings could be used in the 8″ width of lumber, or 1 row of 4″ split rings could be used in the 6″ width of lumber. To keep the number of bolts at a minimum, 4″ connectors are recommended, although in a structure where 2½″ split rings are used in the other joints, it may be best to use them in this joint also.

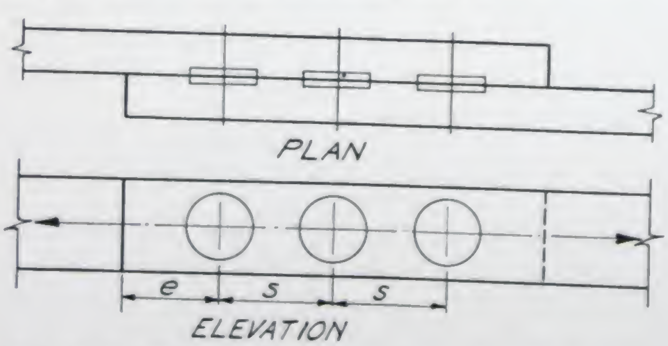


Fig. 15—Two-Member Tension Joint, Loaded Parallel to Grain

Net Section: The net section of a member remaining after boring the bolt holes and cutting the grooves must then be checked to determine if it is adequate; it will usually be found adequate except where lumber of the higher stress grades is designed to nearly its full capacity. Checking net section is readily done by referring to the tables on page 15 of the Manual. In the lower table it will be found that the projected area of one 4″ split ring and bolt in a member 1⅝″ thick is 3.09 sq. in. and in a member 2⅝″ thick is 3.84 sq. in. These values subtracted from the sectional areas of the 2″ x 8″ and 3″ x 6″ lumber sizes leave 9.10 sq. in. and 10.93 sq. in. respectively. To determine then, if the net section remaining is sufficient, the actual number of square inches of net section required is computed from the table of constants, page 15, by multiplying the load to be carried by the constant for Standard loading for Group B Species in material less than 4″ in thickness, which in this case is 14,000 x .00046, or 6.45 square inches. Therefore either the 2″ x 8″ or 3″ x 6″ lumber sizes furnish adequate net section; the choice of size selected for a structure will be determined from the design as a whole.

Connectors Required: Working loads for split rings are given in the table, page 5, of the Manual. A 4″ split ring used in one face of a member 1⅝″ or more in thickness and loaded parallel to grain has a load capacity of 5500 lbs. for Group B Species, and to carry the 14,000 lb. load, 2.55 or 3 split rings are required. The standard spacing and end distance for these connectors for load applied parallel to grain (see Manual, page 4) are 9″ and 7″ respectively. With three bolts spaced 9″ apart and a 7″ end distance, the joint length from the end of one piece to the end of the other piece in the joint is 7″ + 9″ + 9″ + 7″ or 32″.

2. Permissible Reduced Spacings

Assume that it is desirable to shorten as much as possible the spacing between bolts with 4″ split rings in the tension joint discussed above and still carry the 14,000 lb. load.

The total capacity of three 4″ split rings is 3 x 5500 lbs. or 16,500 lbs., but only 14,000 lbs. or 85% of their capacity need be developed. Therefore, instead of the three connectors carrying 300% of the capacity of one connector, they need carry a total of only 255%. Since one connector in a row with reduced connector spacing is assumed to carry 100% capacity, the other two connectors need to develop 77.5% capacity for each. By interpolation of spacing requirements from full spacing of 9″ at 100% capacity to 4⅞″ at 50% capacity, as given in the Manual, or from the loose-leaf Spacing and Distance Chart No. 1 included with this booklet, the spacing may be reduced from 9″ to 7⅛″. The total joint length from the end of one member to the end of the other member in the joint is then 7″ + 7⅛″ + 7⅛″ + 7″ or 28¼″, a reduction of 3¾″ from the length of the joint with full spacing and end distances.

3. Permissible Reduced End Distance

If the end distance only is to be reduced, the percentage reduction may be applied to the end ring up to the maximum of 37.5% allowed, which takes the minimum end distance permitted. Since the permissible reduction for the joint is 45%, it will be seen that the end distance may be reduced to the minimum allowed, namely to 3½" for the 4" split ring. The total joint length is then 3½" for the 4" split ring. The total joint length is then 3½" + 9" + 9" + 3½" or 25".

4. Permissible Reduced Spacing and Reduced End Distances

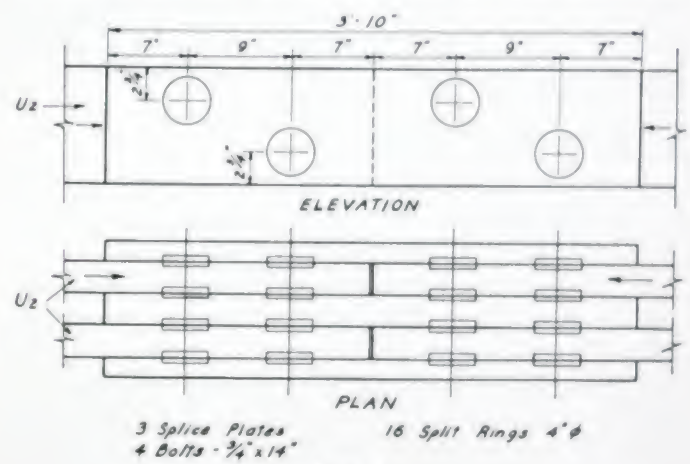
In some cases it may be desirable to reduce both end distances and spacings. Such reductions are made by combining the two systems described under 2 and 3 above. Assuming that the end distances are reduced, the full amount thereby reducing the capacity of the joint by 37.5%, it still leaves 45% — 37.5% or 7.5% which can be applied to reduced spacing. The 7.5% divided by the two bolts with connectors (exclusive of the end bolt) gives 3.75%. By interpolation for reduced spacing, the 3.75% permits the spacing to be reduced from 9" to 8¾". The total joint length would then be 3½" + 8¾" + 8¾" + 3½" or 24½".

Compression Joint—A compression joint may be designed similarly to a tension joint with all the load transferred by connectors or it may be designed with part of the load carried by connectors and part carried by end bearing of the members with a metal plate fitted snugly between them. It is frequently found more practical to design a compression joint with connectors carrying the entire load, since splice plates provide the necessary stiffness for the members joined and a minimum amount of labor is required to fabricate the joint. The design of the joint with part of the load carried by direct end bearing offers a convenient solution, on the other hand, when using comparatively large members with limited space for splicing and where lateral stiffness is provided by means other than the joint itself, such as being placed near or in a joint with lateral bracing. Compression joints in the top chords of bridge trusses offer typical examples where this method of designing joints is found. As much as 100% of the bearing stress of the members may be developed by end bearing, provided a metal bearing plate is placed between end of members and fabrication is accurate. Normally, however, not over 50% to 75% is practical since the joint must be held together by some means. This is handled advantageously by connectors and one or more splice plates for holding the joint in line and for carrying a portion of the load; the remaining portion of the load being transferred by end bearing of members against a snugly fitted bearing plate.

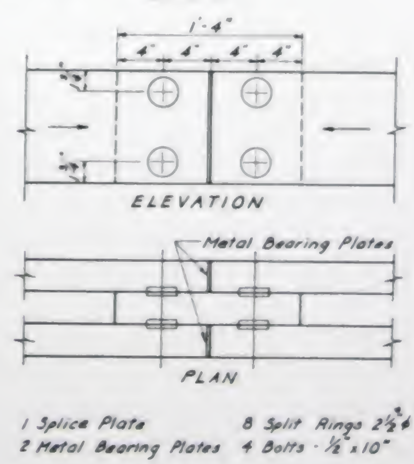
Another system sometimes used is that of filling with concrete the space enclosed by the metal gusset plates and the ends of the members, the concrete forming end bearing for all the members entering the joint.

The joint in the top chord member U2, Fig 14, will serve to explain the two design systems. Reference to the previous text, describing the method of finding the size of the top chord members, will show that the top chord takes two pieces of lumber 3" x 10", nominal. For one condition assume that the entire load of 35,100 lbs. is carried by connectors, and for the other condition, that part of the load will be transferred by end bearing of the chord members. See Fig. 16, Joint A and Joint B respectively.

With the entire load transmitted by splice plates, their combined sectional area must equal at least the total sectional area of the members joined. A center piece 2⅝" thick and two side pieces 1⅝" thick provide the necessary section. Both 2½" and 4" diameter rings



Joint A



Joint B

Fig. 16—Details of Compression Splice with (A) Entire Load of 35,100 lbs. Carried by Connectors, and (B) 31% of the Load Carried by Connectors and 69% Carried by End Bearing of Members Against Metal Plates

will be considered since these are the most common sizes used in this type and size of truss. The 35,100 lbs. load for Group B Species requires 12.3 split rings 2½″ in diameter each side of the joint, or 6.5 split rings 4″ in diameter. Twelve 2½″ split rings each side of the joint will safely carry the load and will fit into the joint conveniently. It will be necessary to use 8 split rings 4″ in diameter to keep the joint symmetrically loaded. The cost of labor for installation of rings of either size will be about equal, the single apparent advantage of one ring size over the other being that the 2½″ split rings take slightly shorter splice plates. The joint in Fig. 16-A shows 4″ split rings with end distances and spacings for full load capacity for the rings. The rings are placed off the center line to provide more even distribution of load to the members.

Fig. 16-B shows the same joint with the same load transmitted and member sizes as given in Fig. 16-A, but with bearing plates between the ends of the members. The outside splice plates are omitted and 2½″ split rings are used instead of the 4″ split rings. In this joint 31% of the load is carried by the rings and 69% is transferred through the abutting ends of the compression members.

Load at Angle to Grain

Most structural joints have members entering at an angle with reference to other members and therefore produce bearing at an angle of load to grain. The several examples of joints in the following discussion are all of this type with each one bringing out certain design features.

Chord Members Placed Between Web Members—This is a type of joint commonly used in the design of pitched roof trusses. By making the compression member the same thickness as the splice plates used between the chord members, a comparatively low l/d ratio is secured and furthermore, the compression member will also fit between the chord members. The tension members in the joint may be thinner since their l/d ratio is not a factor and further, since they are placed outside the chord members.

A joint of this type is formed by members L1, L2, V2, and D3 of Fig. 14, and is detailed to larger scale in Fig. 17.

In this joint, each of the two tension members D3 exerts its load of 5000 lbs. (total 10,000 lbs.) at 45° to the grain in the two lower chord members. At this angle of load to grain a 4″ split ring in lumber 2⅝″ thick of Group B Species develops a safe working load of 4590 lbs. Two of these rings, one between each chord member and each diagonal, with a ¾″ bolt will develop 9180 lbs. But this is not sufficient to carry the 10,000

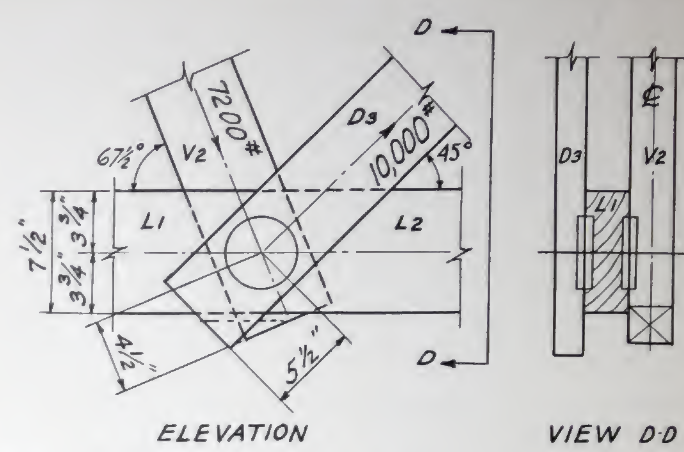


Fig. 17—Detail Drawing for Lower Chord Joint with 4″ Split Ring and 1″ Diameter Bolt

lb. load, and therefore to increase the load capacity without increasing the size of the members to accommodate an extra bolt or more rings, a 1″ bolt is used in place of the ¾″ bolt, thereby increasing the capacity of the connectors by 7% (See Supplement No. 6, footnote for table of split ring loads), giving a total of 9800 lbs. This being only 2% under the full connector capacity required, the joint is adequate to meet good design requirements. Since the load in the tension member is acting upward at 45°, the edge distance in the chord member must be 3¾″ on the upper side of the ring where its outside surface is in compression against the wood.

The end distance required for the 4″ split ring in tension member D3, where the load is acting parallel to grain, is determined by the load it carries. The 4590 lb. load capacity of the 4″ split ring increased 7% for the 1″ bolt, equals 4910 lbs. or 85% of the capacity of a ring with a one-inch bolt loaded at 0° to grain. This percentage capacity permits a 5½″ end distance.

Member V2 transmits load to the lower chord at 67½° to grain, at which angle the load per ring with a 1″ bolt is 4185 lbs. plus 7% or 4480 lbs., and for two rings is 8960 lbs. This is greater than the 7200 lbs. load necessary. The load in this web-member is acting downward so that the compression side of the ring is toward the bottom edge of the chord, therefore this edge distance as well as that for the upper side must be 3¾″, making a member with a 7½″ face necessary. It is this joint in the bottom chord which requires the widest face width and therefore determines the width of the member.

The standard end distance for a compression member with a 4″ split ring is 5½″, but since only 80% of the load need be developed in member V2, the distance may be reduced to 4½″. The ends of both members V2 and D3 may be sawed off parallel to the edge of the bottom

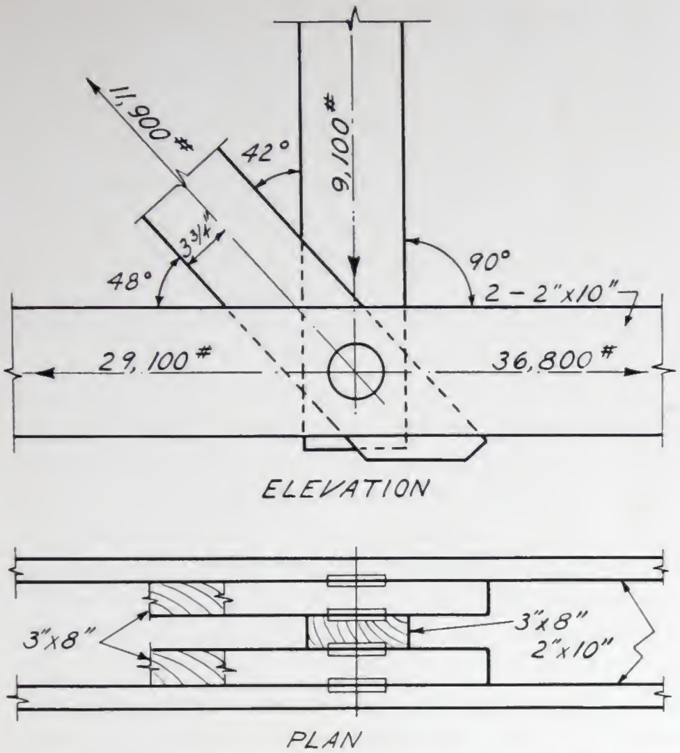


Fig. 18—Diagonals Between Vertical and Horizontal Members

chord about ½" below it as shown by the dotted lines, Fig. 17, which still leaves sufficient end distance.

Chord Member Placed Outside of Web-Members—The lower chord joint of a flat top Pratt truss shown in Fig. 18 has the diagonal members placed between the vertical and the horizontal members. Advantages gained by this arrangement as compared to the diagonals being placed outside the chord members, are that (1) the diagonal members which must transmit the greatest load have two faces into which connectors may be placed, whereas, if they are located outside the chord members, connectors could be placed in only one face of each diagonal; (2) the bottom chord which is in tension does not need to be the same thickness as the top chord to assemble the truss; (3) the working load per connector between the vertical and diagonal with a 42° angle is considerably greater than it would be between the vertical and bottom chord with a 90° angle of load to grain.

An analysis of the joint in Fig. 18 with axial loads in the members, shows that where three overlapping members in a joint come together at different angles, it is the center member with connectors in both faces which has the load at an angle to grain; the two side pieces being stressed only parallel to the grain. The side pieces (in this case being the horizontal chord and vertical web-member) must deliver their loads in the direction of their respective axes thereby introducing stresses in the diagonal piece, which combine to produce the resultant force equal to the load carried by the diagonal member. Since the compression side of the ring in one face of the diagonal member applies to one edge, and the compression side of the ring in the opposite face applies to the opposite edge, due to the direction of loads, the diagonal must have a 3¾" edge distance on each side resulting in a member with a face width of 7½".

The vertical member in Fig. 18 with a 9100 lb. load to transfer to the two diagonal members at 42 degrees, will take two 4" split rings, one on either side, to carry the load, since at this angle, in Group B Species, and for lumber 2⅝" thick, one 4" ring will carry 4645 lbs. and two will carry 9290 lbs. The two horizontal chord members must transmit a total of 7700 lbs. to the diagonal members (36,800-29,100) at an angle to grain of 48°. Under the conditions presented, one ring will carry 4540 lbs. and two will carry 9080 lbs., which is more capacity than necessary to carry the 7700 lb. load. Therefore, since the 4" split rings are sufficient to transfer the vertical and horizontal loads, they must also be sufficient to carry the load in the diagonal members.

Heel Joint—Fink Truss—The heel joint in the Fink Truss shown in Fig. 14 is simplified by extending the top chord members to bear on the support, thereby transmitting directly to the support that portion of the load acting as the vertical component and eliminating the necessity of transferring this portion of the load by connectors through the bottom chord. (See Fig. 19.)

In the design of the heel joint, it is apparent that split rings, toothed rings, claw plates or shear plates may be used in this timber-to-timber connection. Split rings, however, will be found to be most suitable since they carry more load than toothed rings of comparable size; also split rings are more easily installed than claw plates, no pressure being necessary to seat them; and

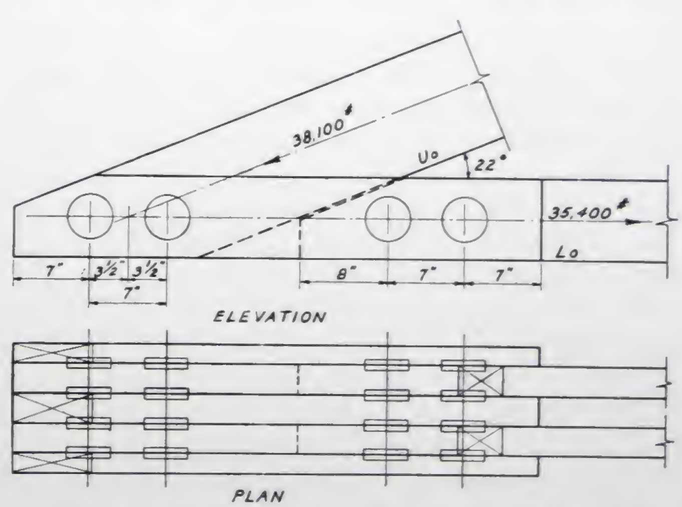


Fig. 19—Heel Joint of Fink Truss with Top Chord Members Extending to Support

split rings are less expensive to use than either claw plates, or flange shear plates because 2 plates must be used to make a single timber-to-timber connection compared to one split ring. As previously noted it is usually more efficient to use the larger connector sizes since they keep the number of bolts and connectors at a minimum and facilitate fabrication. If the most appropriate connector size is not immediately evident for the joint, the number of connectors required for each size may then be computed and one selected which fits most advantageously into the members. The loads for split rings for the heel joint in Fig. 19 are dependent on the 22° angle of load to grain bearing of the connectors in the top chord, on the 2⅝″ lumber thickness as determined previously and on the Group B Species of lumber. It is assumed that the truss will be used in a location where the lumber will reach an air-dry condition and, therefore, the moisture content of the lumber need not be considered as influencing split ring loads. Based on the above conditions, the safe load for a 2½″ split ring is 2640 lbs., for a 4″ split ring the safe load is 5005 lbs. and for a 6″ split ring the safe load is 6440 lbs.

The 35,400 lb. horizontal load which must be transferred to the bottom chord will take 13.4 split rings 2½″ in diameter, 7.1 split rings 4″ in diameter or 5.5 split rings 6″ in diameter. With four contacting faces into which connectors must be placed to keep the joint symmetrically loaded, the number of rings must be rounded off to sixteen 2½″ rings, eight 4″ rings or eight 6″ rings. This number of rings for each size can be located in the heel joint to carry the load. It will be noted, however, that the 4″ ring size is the most efficient, in that the number of connectors actually required in the joint approaches nearest the number which is recommended for a symmetrically loaded joint. Also, the 4″ rings take only 2 bolts and 8 connectors as compared to 4 bolts and 16 rings of the 2½″ size. If 6″ rings were used they still would require 2 bolts and, in addition to being less economical for this joint, they would be found neither necessary nor convenient for any other joint in the truss, therefore the 4″ rings are recommended. A further consideration in the design of a joint where more than one bolt with connectors occur is that where the full capacity of the connectors is not developed as in this joint it may be desirable to reduce their spacings and distances. The rule for reduced spacing as discussed under "Spacing" may be applied, since the full capacities of the 4″ rings are not developed. According to the rule the 4 rings on one bolt will carry 400% of one ring capacity, or 400% x 5005 lbs. = 20,020 lbs. This amount subtracted from 35,400 lbs. leaves 15,480 lbs. for the 4 connectors on the second bolt or 77% of their rated load. To develop 77% load capacity, of the connectors in the second bolt, spacing between bolts may be reduced to 7″ as determined by interpolation from design data given in the Manual or from the loose leaf Spacing and Distance Charts. End distance specified for the bottom chord splice plates stressed in tension parallel to grain is 7″, and end distance for the top chord in compression is 5½″ measured parallel to grain. Edge distance, since the angle of load to grain is less than 30 degrees, remains at the 2¾″ minimum for both members and is provided by the members used.

The load to be transferred between the two bottom chord members and the splice plates is parallel to grain, and for the conditions presented a 4″ split ring will carry 5400 lbs. To carry the 35,400 lb. load, 6 rings are required; 8 rings, however, are recommended to maintain a symmetrically loaded joint. The diagonal cut on the end of the bottom chord member through a point giving a 7″ standard end distance, measured along the center line, is not sufficient to provide the necessary 2¾″ distance (equal to edge distance for 4″ split ring) measured perpendicularly from the diagonal cut to the center of the bolt hole. Therefore, the end distance at the center line must be increased. Since the edge distance must be 2¾″, the distance along the center line of the bottom chord is 2¾″ ÷ sine 22° or 7.2″

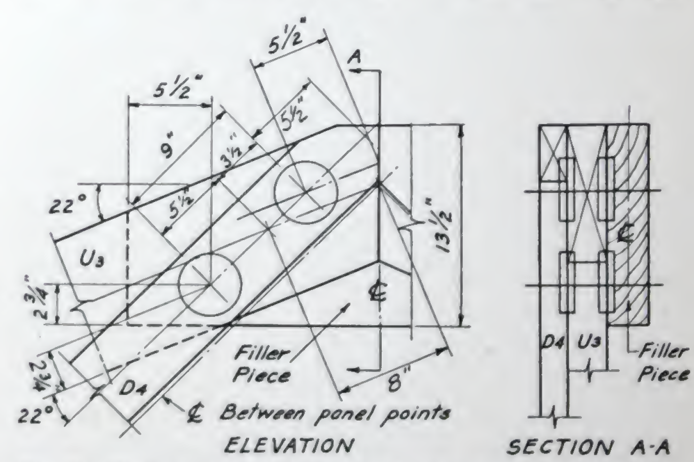


Fig. 20—Detail of Peak Joint with 4″ Split Rings and ¾″ Bolts

required. In practice, the end distance probably would be scaled and rounded off to 7½″ or 8″. Spacing of the connectors may be taken as full 9″ as specified for full load capacity where there is sufficient area to place them along the chord, or the spacing may be reduced following the practice discussed under the Tension Joint. Spacing of connectors is shown as 7″ in Fig. 19 to correspond to the 7″ end distance of the splice. A check of the net section required for the members, using the same procedure as that explained for the Tension Joint will show that sufficient section is provided by the size of members used.

Peak Joint—The design of the peak joint in the Fink Truss, Fig. 14, demonstrates how two bolts with connectors may be located to accommodate three members entering the joint when the angle of load to grain is less than 30 degrees, for each two contacting members. In this joint as detailed in Fig. 20, the 15,000 lbs. must be transferred from the two diagonals D4 to the two chord members U3 and the 20,000 lbs. horizontal component must be carried through the splice to the other half of the truss. (This horizontal component is equal to the tension in the bottom chord at the center of the truss. See Fig. 14.) In this joint, connectors are located in the two overlapped areas between the chord members and diagonals, also between chord members and the filler piece. The larger load of 20,000 lbs. should be considered first in the design of the joint, since connectors placed on the bolts used for the larger load will quite probably provide spacings and distances to develop the smaller load. Part of the thrust between the two half trusses could be carried by end bearing of the top chord members, in which case a fitting job for the metal bearing plates would be required. However, the two half sections of the roof truss must be securely attached at this joint and, regardless of the means for absorbing the thrust, a splice is necessary and this can perform the functions of both a tie and a splice to carry the load.

In the top chord members a 22° angle of load to grain is made by the splice piece and also by web-members D4. At this angle and with standard distances and spacings, the load per connector is 5005 lbs. With full distances and spacings therefore, 4 connectors on two bolts will carry the 20,000 lb. load to be transferred between the two chord pieces U3 and the filler piece. Since this part of the joint is in compression, the end distance for 4″ split rings must be 5½″. This end distance for piece U3 then determines the gauge line for the end connector, the gauge line being parallel to the end cut and measured parallel with the grain in the chord. The required 9″ spacing and 2¾″ edge distances for all members joined must also be met. In order to locate the two bolts with connectors within the limits of the diagonal D4 and still have the specified edge distance for connectors in the chord member, it is necessary to offset the diagonal so that the intersection of the center lines of the diagonal and chord members is 8 inches to the left of the panel point. The slight eccentricity introduced into the joint by offsetting diagonal D4 is of little importance and may be neglected. The filler piece must be of sufficient width and length to provide the specified edge and end distances for connectors in that piece. A splice piece 14″ wide will provide the necessary width and the length may be extended to meet the requirements.

The four 4″ split rings between the chord and web-members must be investigated to determine if they develop sufficient load capacity to carry the 15,000 lb. load. Since web-members D4 are in tension they take an end distance of 7″ to develop full load capacity for the connectors. The spacing of connectors is the required 9″, therefore, the only reduction in load capacity is that due to the reduced end distance from 7″ to 5½″ (See Fig. 20). It will be noted that for these diagonal members, there are two diagonal end cuts and therefore, the end distance is not measured to the extreme end of the member but to a point a short distance from the end which gives approximately the required shear area for the connector. A reduction of 1½″ in end distance reduces the load capacity 17% (see charts). Therefore, two connectors with full load carry 10,010 lbs. and the other two connectors carry 83% of 10,010 lbs. or 8310 lbs., which totals 18,320 lbs. or 3,320 lbs. in excess of the 15,000 lbs. necessary to be developed.

DESIGN OF MEMBERS WITH COMBINED BENDING AND AXIAL LOADS

Combined axial and bending stresses occur in chords of trusses on which load is applied between panel points, rafters with collar beams, or columns supporting eccentric load, such as a column with an attachment for supporting a load on one side. When computing the stresses in members with combined loads, the resulting stress must necessarily be a combination of stresses due to each type of loading. Since the allowable tension stress in a timber differs from the allowable compression stress, one formula is necessary for combined *tension* and bending and another formula is necessary for combined *compression* and bending.

Combined Axial Tension and Bending

Allowable tension stresses and allowable bending stresses are the same for specific grades of lumber (See Supplement No. 1) and, therefore, the total allowable stress in the member is the sum of the two. A member with this type of combined loading (see Fig. 21) must then be designed so that the sum of the stresses must not exceed the bending or tension stress (f) of the grade of lumber used. This combination of stresses may be expressed by the formula:

$$^{*}\frac{P}{A} + \frac{M}{S} \quad \text{must not exceed the fiber stress in bending (f)} \qquad (1)$$

in which

P = Total axial load in pounds.
A = Area in square inches of cross section of members.
M = Total bending moment in inch pounds.
S = Section modulus of member.
f = Allowable extreme fiber stress in bending in pounds per square inch.

* See also Wood Structural Design Data, page 49-B.

An example of combined tension and bending in a member is the bottom chord of a Fink truss when the chord is uniformly loaded by ceiling joists. Chord members of a truss are normally continuous over 2 or more panel lengths and consequently develop continuous beam action. In a continuous beam having equal spans and uniformly distributed loading, the moment at the support next to its end determines the load capacity of the beam in bending. When the beam is continuous over three or more spans, this limiting moment is approximately $\frac{Wl}{10}$ and when the beam is continuous over two spans or is a simple beam, the limiting moment is $\frac{Wl}{8}$. Therefore, the moment which determines the load capacity in bending of the chord is usually that at the panel point next to the end of the

Fig. 21—Combined Axial Tension and Bending

truss or at a panel point near a splice; in this instance since we are assuming that the beam is continuous over three or more panels the maximum moment is $\frac{Wl}{10}$. With reference to a span in which a splice occurs, it is common practice to use the same bending moment $\frac{Wl}{10}$ for this span also.

Example:

For an example showing the method used in determining the stress in a member with combined tension and bending assume:

- Truss spacing 16 ft. on centers
- Panel length 10 ft.
- Total live and dead load 40 lbs. per sq. ft.
- Tension in chord member 25,000 lbs
- Tentative size of chord member 5½" x 11½"
- Fiber stress in bending and in tension 1200 lbs. per sq. in.

Solution:

The adequacy of a proposed member is determined by computing the fiber stress (f) as follows:

Substituting in the formula:

$\frac{P}{A} + \frac{M}{S}$ must not exceed the fiber stress in bending (f)

$$\frac{P}{A} + \frac{\frac{Wl}{10}}{\frac{bh^2}{6}} = \frac{25{,}000}{63.25} + \frac{76800}{121.2} = 395 + 634 = 1029\text{f}$$

Since the total 1029f is less than the 1200 lbs. per square inch fiber stress for the timber grade proposed, the 6" x 12" piece is of adequate size.

COLUMNS (COMPRESSION MEMBERS) WITH COMBINED END LOAD, SIDE LOAD AND ECCENTRICITY

In the design of wood columns it is customary to assume that they have pin ends even though they may be square cut at the ends or are continuous through points of lateral support.

Columns Laterally Supported

Lateral support as it pertains to application of the following formulas implies that the member is supported in a direction perpendicular to the direction of the side load in such a manner that the dimension of the

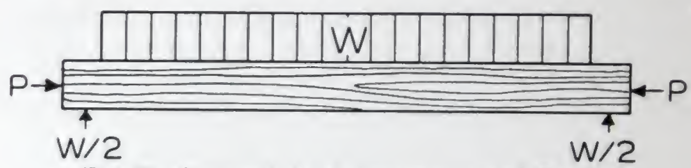


Fig. 22—Combined Axial Compression and Bending
In the Accompanying Discussion It Is Assumed that the Member Is Restrained from Buckling in a Direction Perpendicular to the Side Load.

member in the direction of the side load (rather than the other cross sectional dimension) determines the length-breadth $\left(\frac{l}{d_b}\right)$ ratio to be used for the column.

For rectangular columns with combined end loads, side loads (See Fig. 22) and eccentricity the direct compressive stress in pounds per square inch should not exceed 'Q' computed from the formulas given herewith.* (Note: In the following general formulas (2) and (3), the effect of end loads, side loads and eccentricity are all considered. If some of these types of loads are not present, some of the symbols become zero and the formulas will be simplified somewhat. When there is no eccentricity 'e' in the formulas becomes zero, when side loads are not proportional to end loads 'k' becomes zero, and when side loads are proportional to end loads 's_b' becomes zero.)

SYMBOLS USED

f = Allowable extreme fiber stress in bending in pounds per square inch.

l = Length of span or column in inches.

d_b = Dimension of a rectangular column in the direction of the side load.

e = Eccentricity in inches.

$s_b = \frac{M}{S}$ = Unit bending stress due to side loads.

* For derivation of these formulas see article "Formulas for Columns with Side Loads and Eccentricity" by J. A. Newlin, Specialist in the Mechanics of Timber, Forest Products Laboratory, Madison, Wis., and published in Building Standards Monthly, December, 1940.

k = The ratio of the unit bending stress due to side loads to the unit compressive stress due to end load when side load is proportional to end load.

c_b = Allowable stress in compression parallel to the grain in pounds per square inch that would be permitted for the column if axial compression stress only existed, i. e., the safe working stress for the appropriate $\frac{l}{d_b}$ ratio.

Q = Allowable stress in compression parallel to grain in pounds per square inch that is permitted for the column when side and/or eccentric loads are combined with end loads.

Columns with $\frac{l}{d_b}$ = 20 or more

$$Q = \frac{f + c_b\left(1 + \frac{15}{2}\frac{e}{d_b} + k\right)}{2} - \sqrt{\left\{\frac{f + c_b\left(1 + \frac{15}{2}\frac{e}{d_b} + k\right)}{2}\right\}^2 - c_b\,(f - s_b)} \tag{2}$$

Columns with $\frac{l}{d_b}$ = 10 or less

$$Q = \frac{c_b(f - s_b)}{f + c_b\left(\frac{6e}{d_b} + k\right)} \tag{3}$$

Columns with $\frac{l}{d_b}$ between 10 and 20

Assume a straight line variation of stress with $\frac{l}{d_b}$ between the stress at $\frac{l}{d_b} = 10$ and $\frac{l}{d_b} = 20$, using 'e', 'k', and 's_b' as found for the particular member, but using 'c_b' for $\frac{l}{d_b} = 10$ in the one formula and for $\frac{l}{d_b} = 20$ in the other formula.

(Note: The formula for $\frac{l}{d_b} = 20$ or more may be used to an $\frac{l}{d_b} = 10$ to check the adequacy of a member, since the values for 'Q' by this method will be somewhat lower than the requirement considering a straight line variation in stress between $\frac{l}{d_b} = 10$ and $\frac{l}{d_b} = 20$).

The values of 'e', 'k', and 's_b' may vary widely and each independent of the other. Any of them may be equal to zero or they may act in opposite directions.

The loads shall, however, all be assumed to act in the same direction unless it can be definitely established that under all conditions of loading which may come upon the structure, they will act in opposite directions.

Formulas for Specific Loading Conditions

The following formulas for specific loading conditions are deduced from the general formulas above in accordance with the notes accompanying them. The two previous formulas, (2) and (3), are also repeated here.

Columns with $\frac{l}{d_b}$ = 10 or less

Combined End and Side Load

$$Q = \frac{c_b\,(f - s_b)}{f} \tag{4}$$

Eccentrically Loaded Columns

$$Q = \frac{c_b\, f}{f + c_b\left(\frac{6e}{d_b}\right)} \tag{5}$$

Side Loads Proportional to End Loads

$$Q = \frac{c_b\, f}{f + c_b\, k} \tag{6}$$

Combined End Load, Side Loads and Eccentricity

$$Q = \frac{c_b\,(f - s_b)}{f + c_b\left(\frac{6e}{d_b} + k\right)} \tag{3}$$

Columns with $\frac{l}{d_b}$ = 20 or more

Combined End and Side Load

$$Q = \frac{f + c_b}{2} - \sqrt{\left\{\frac{f + c_b}{2}\right\}^2 - c_b\,(f - s_b)} \tag{7}$$

Eccentrically Loaded

$$Q = \frac{f + c_b\left(1 + \frac{15e}{2d_b}\right)}{2} - \sqrt{\left\{\frac{f + c_b\left(1 + \frac{15e}{2d_b}\right)}{2}\right\}^2 - c_b\, f} \tag{8}$$

Side Loads Proportional to End Loads

$$Q = \frac{f + c_b\,(1 + k)}{2} - \sqrt{\left\{\frac{f + c_b\,(1 + k)}{2}\right\}^2 - c_b\, f} \tag{9}$$

Combined End Load, Side Load, and Eccentricity

$$Q = \frac{f + c_b\left(1 + \frac{15e}{2d_b} + k\right)}{2} - \sqrt{\left\{\frac{f + c_b\left(1 + \frac{15e}{2d_b} + k\right)}{2}\right\}^2 - c_b\,(f - s_b)} \tag{2}$$

Columns with $\frac{l}{d_b}$ between 10 and 20

The stress should be assumed to vary as a straight line between these limits as previously noted for permissible loads per square inch computed by the formulas for $\frac{l}{d_b} = 10$ or less and for $\frac{l}{d_b} = 20$ or more. In practice

since the value for 'Q' computed by the formula for $\frac{l}{d_b} = 20$ or more will be found to be only slightly less than the value found by a straight line interpolation between the two formulas, it will usually be necessary to apply only the one formula, namely the one for $\frac{l}{d_b} = 20$ or more.

An example of loading which introduces combined axial compression and side loading in a member is that found in the top chord of a roof truss with roof joists spaced between panel points. The effect of continuous beam action over one panel point applies here as in the case of a member subjected to combined tension and bending; the bending moment being computed with the same formula, namely, $\frac{Wl}{10}$. With reference to a span in which a splice occurs, it is common practice to use the same bending moment $\frac{Wl}{10}$ for this span also.

Example: $\left(\frac{l}{d_b} = 10 \text{ or less}\right)$

As a specific example for determining the adequacy of the top chord member of a truss with combined compression and bending assume the following values:

Truss spacing, 16 ft. on centers
Uniformly loaded top chord
Panel length (flat roof) 10 ft.
Total live and dead load, 40 lb. per sq. ft.
Compression load in top chord member, 25,000 lbs.
Tentative size of chord member, 5½″ x 13½″
Allowable fiber stress in bending, 1,200 lbs. per sq. inch
Allowable compression stress, 900 lbs. per sq. inch
Modulus of elasticity (E), 1,600,000 lbs. per sq. inch.

Solution:

For the above conditions and with the members restrained from buckling in a lateral direction in the plane of the roof, the $\frac{l}{d_b}$ of the member with 'd_b' the dimension of the member measured in the direction of the side load, is $\frac{120}{13.5}$ or 8.9. Since this ratio is less than 10, and since the side load is proportional to the end load and further, since there is no eccentricity, the following formula applies:

$$Q = \frac{c_b f}{f + c_b k} \qquad (6)$$

Substituting values for the symbols in the formula

$f = 1{,}200$ lbs. per sq. inch
$c_b = 900$ lbs. per sq. inch (See page 4, Supplement No. 4)

$$k = \frac{\frac{M}{S}}{\frac{P}{A}} = \frac{\frac{40 \times 16 \times 10 \times 120}{10 \times 167.06}}{\frac{25{,}000}{74.25}} = \frac{460}{337} = 1.37$$

Then

$$Q = \frac{900 \times 1{,}200}{1{,}200 + 900 \times 1.37} = 444 \text{ lbs. per square inch}$$

Since the permissible computed load of 444 lbs. per sq. inch is greater than the actual column load of 337 lbs. per sq. inch as found for determining "k", the column is adequate. A similar check will show that a 5½″ x 11½″ member would be inadequate.

Example: $\left(\frac{l}{d_b} \text{ ratio between 10 and 20}\right)$

For an example illustrating the application of the formula where the $\frac{l}{d_b}$ ratio is between 10 and 20, assume:

Truss spacing 12 ft. on center
Uniformly loaded top chord
Total live and dead load 30 lbs. per sq. ft.
Panel length (flat roof) 12 ft.
Compressive load of top chord 20,000 lbs.
Tentative size of top chord 5½″ x 11½″
Allowable fiber stress in bending f 1,200 lbs. per sq. ft.
Allowable compressive stress c 900 lbs. per sq. inch
Modulus of elasticity E 1,600,000 lbs. per sq. inch
Top chord is restrained laterally.

Solution:

The $\frac{l}{d_b}$ ratio of the member with 'd_b' the dimension of the side of the member measured in the direction of the side load is $\frac{144}{11.5}$ or 12.5; this being a ratio between 10 and 20 with side loads proportional to end loads, and no eccentricity the two following formulas apply with a straight line interpolation between the two results secured to give the permissible unit safe loads.

$$\frac{l}{d_b} = 20 \text{ or more}$$

$$Q = \frac{f + c_b(1+k)}{2} - \sqrt{\left\{\frac{f + c_b(1+k)}{2}\right\}^2 - c_b f} \qquad (9)$$

$$\frac{l}{d_b} = 10 \text{ or less}$$

$$Q = \frac{c_b f}{f + c_b k} \qquad (6)$$

If it is found by substituting in formula (9) that the load capacity developed in the member is sufficient to meet the requirements, no further computations are

necessary. However, if the capacity of the member is slightly lower than that required, it may then be desirable to compute the stresses by each of the formulas $\frac{l}{d_b} = 20$ or more and $\frac{l}{d_b} = 10$ or less and then determine the stress by a straight line ratio as explained previously. The stress thus determined will be slightly greater than that determined by formula (9) and may be sufficient to develop the necessary load. If this new stress is still insufficient, it then becomes necessary to investigate a larger sized member.

Substituting values for the symbols in the formula (9):

$f = 1{,}200$ lbs. per sq. inch

$c_b = 887$ lbs. per sq. inch (See page 4 Supplement No. 4)

$$k = \frac{\frac{M}{S}}{\frac{P}{A}} = \frac{\frac{30 \times 12 \times 12 \times 144}{10 \times 121.23}}{\frac{20{,}000}{63.25}} = \frac{514}{315} = 1.63$$

Then

$$Q = \frac{1{,}200 + 887(1 + 1.63)}{2} - \sqrt{\left\{\frac{1{,}200 + 887(1 + 1.62)}{2}\right\}^2 - 887 \times 1{,}200}$$

$$= 1{,}765 - 1431 = 334 \text{ lbs per sq. inch}$$

Since the permissible end load stress is 334 lbs. per sq. inch and only 315 lbs. per sq. inch need be developed, the member is, therefore, of adequate size.

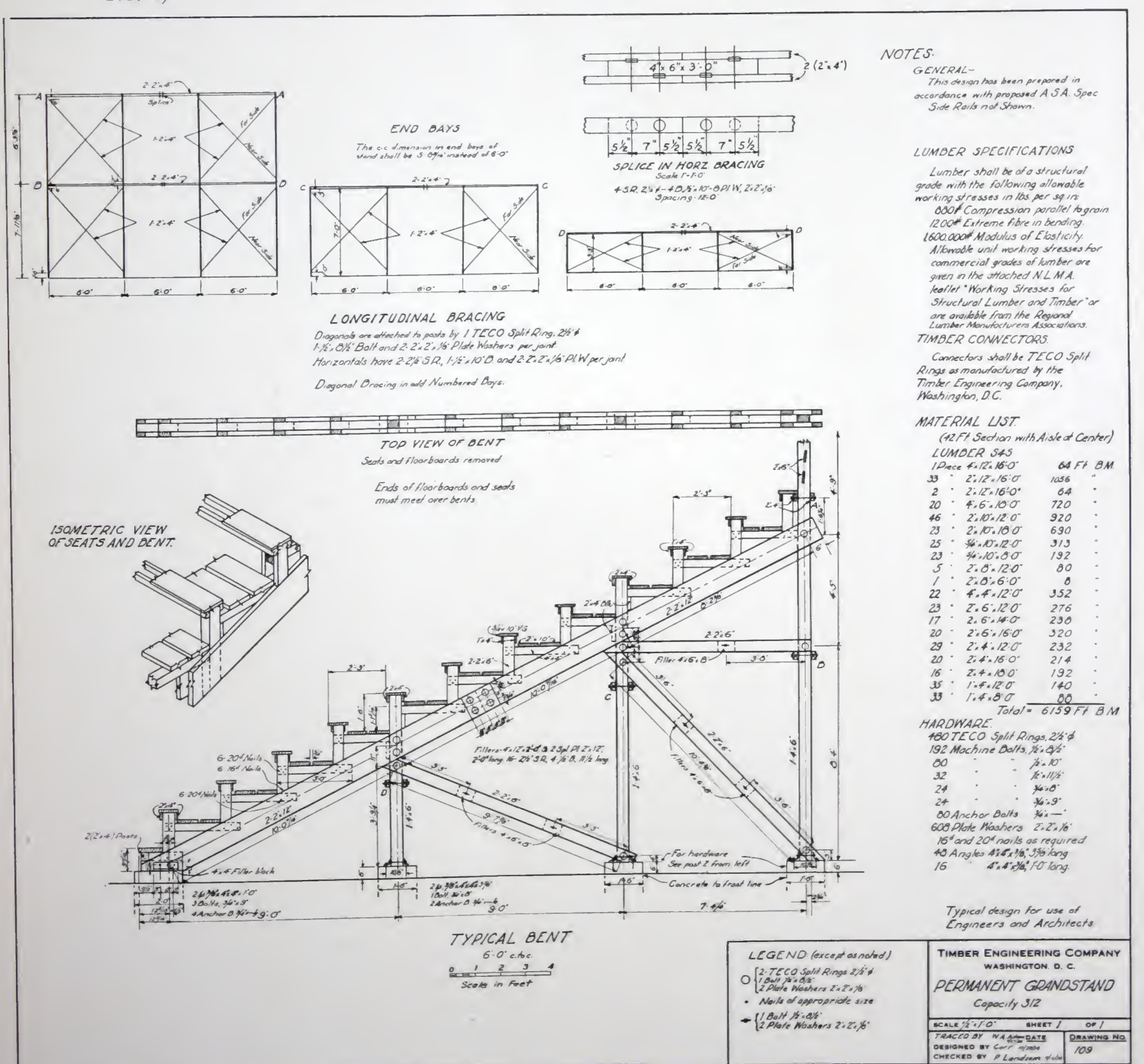


Plate III

Lumber and Connector Literature

Timber Engineering Company,

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Wood Structural Design Data is a 300-page book published by The National Lumber Manufacturers Association. It contains complete tables of safe loads for wood joists, beams and columns covering the full range of commercial sizes, species and grades, and of the spans and heights ordinarily used. The book contains in addition much information on the grading of structural timber, properties of sections, board feet per linear foot and other data useful to designers. Price $1.00 per copy; with following supplements $1.25, U. S. postage paid.

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- Supplement 1—Working Stresses for Structural Lumber and Timber.
- Supplement 2—Bolted Wood Joints—safe loads.
- Supplement 3—Maximum Spans for Joists and Rafters.
- Supplement 4—Wood Columns—safe loads.
- Supplement 5—Wood Trusses—stress coefficients, length coefficients and angles.
- Supplement 6—Manual of Timber Connector Construction.
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